SELECTIVE WEAKENING RETROFIT FOR EXISTING R.C. STRUCTURES – CONCEPT, VALIDATION AND DESIGN EXAMPLE

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ABSTRACT

A summary of the current research towards the development and validation of a counter-intuitive seismic retrofit strategy for non-ductile reinforced concrete (RC) frames, termed as selective weakening (SW) retrofit, is herein presented. The SW retrofit is conceived with the aim for wide-implementation, economical, and non-invasive structural retrofit solution for non-ductile (pre-1970s) RC frame structures. Contrary to the misconception that seismic retrofit must involved strengthening (force-based approach) SW retrofit relies upon targeted weakening of structural elements in order to achieve a ductile failure mechanism, thus explicitly enforcing capacity design philosophy within a displacement-based retrofit strategy. In this research, the SW retrofit is implemented to RC frames and more specifically to exterior beam-column (b-c) joints. The bottom longitudinal reinforcements of the beam are cut at the interface with the column and/or external horizontal pre-stressing is applied to the joint. A more desirable inelastic mechanism can be attained within the b-c connection, leading to improved global seismic performance for the RC frame. Experimental validations of the SW retrofit solution of exterior b-c joints are summarized to complement the conceptual development of the SW retrofit. Insights from quasi-static tests on nine 2/3-scaled exterior b-c joint sub-assemblies (with and without retrofit) are discussed in respect to different retrofitting and as-built parameters. Lastly, a simple hand-calculation retrofit design procedure for a SW retrofit on b-c joints is presented. The results demonstrate the viability of such a simple but structurally efficient seismic rehabilitation strategy.

Introduction

The seismic deficiencies of poorly detailed non-ductile reinforced concrete (RC) moment-resisting frames are widely recognized and researched. In particular, beam-column (b-c) joints are shown to be critical weakness in these structures, typically leading to structural collapse or irreparable damages as observed in the field (Park et al, 1995, Holmes et al, 1996) or in the laboratory (Bracci et al, 1995, Calvi et al, 2002). In brief, the poor behaviour of older RC frame construction can be attributed to: the inadequate shear reinforcement in joint region, the poor bond properties of plain round bars reinforcement, the deficient anchorage details into the joint region and the lack of capacity design consideration.

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In resolving the seismic deficiency of non-ductile RC frames, various seismic rehabilitation solutions have been proposed in the past and implemented with success for b-c joints (fib, 2003, NZSEE, 2006, ASCE-SEI-41-06, 2007). However, this research is motivated by the need for an economical, low-invasive and low-technology structural retrofit solution for wide implementation. Contrary to the common misconception that seismic retrofit must involved strengthening (within force-based seismic engineering paradigm), selective weakening retrofit relies upon targeted weakening of structural elements in order to achieve a ductile failure mechanism, thus explicitly enforcing capacity design philosophy within a displacement based retrofit strategy.

Adopting the 'structural weakening' suggested by FEMA-356 (2000), the research at the University of Canterbury has extended the concept to a "selective" weakening (SW) retrofit strategy (Pampanin, 2006) and implemented it to shear-deficient structural walls (Ireland et al, 2007) as well as to deficient hollowcore floor-seating connections (Jensen et al, 2007). In this research, the focus is on structural intervention on the exterior b-c joints of RC frames. By selectively weakening the beams by cutting the bottom longitudinal reinforcements and/or adding external pre-stressing to the b-c joint, a more desirable inelastic mechanism can be attained, leading to improved global seismic performance. In addition, a partial-to-full SW retrofit intervention (Pampanin, 2006) can be adopted to achieve a range of performance limit states: collapse-prevention to limited damages, within a performance-based retrofit approach. In partial SW retrofit – only limited intervention such as only beam-weakening-only or exterior b-c retrofit is required in order to achieve collapse prevention.

In this contribution, the authors are presenting the summary of research – in terms of the conceptual development, experimental validation and design procedure – related to the SW retrofit for exterior b-c joints of RC frame. It summarizes some of the previous work on SW retrofit (Kam et al, 2008, 2009a, 2009b) with the extension on the experimental validation and the design procedure. This research is part of a national research program on the development of seismic retrofit solutions for multi-storey buildings in New Zealand (FRST Retrofit, 2009).

Concept of Selective Weakening for Retrofit

Existing retrofit strategies and techniques for RC Frame

Global or local strengthening (Figure 1a) has been and still remains the most popular retrofit strategy, particularly when dealing with ordinary importance RC buildings. While adding brace frames or shear walls may certainly lead to a more structurally efficient super-structure (irrespectively of the suggestion for low-invasiveness promoted by the architects and highly desired by the occupants), a proper engineering evaluation of the actual consequences of the overall new structural scheme is crucial. Such a strengthening-only global retrofit might in fact generate excessive damage, if not failure, elsewhere within the overall structural system such as the foundation, whose strengthening costs and effort are definitely not negligible.

Local strengthening of critical elements and components such as steel, concrete or fiber-reinforced polymers (FRPs) jacketing have also shown tremendous potential, though the labor intensity and invasiveness of these retrofit techniques might still be a deterrent to their widespread application. Alternatively, for high-value and high importance structures, the reduction of seismic demand by means of supplemental damping (Figure 1b) and/or use of base isolation system (Figure 1c) has been regular practice, as these allows higher performance levels while being less intrusive. Again, the issues of cost and time/space invasiveness of these common techniques have been the reasons for their limited application. Conceptually, all these common retrofit strategies are illustrated in Figure 1(a-c) within an Acceleration-Displacement Response Spectrum (ADRS) domain.

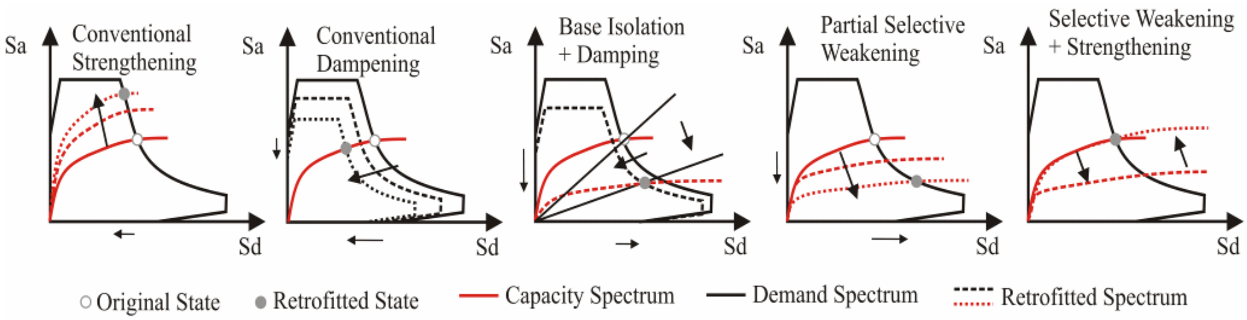

Figure 1. Acceleration-Displacement Response Spectrum (ADRS) illustration of different retrofit strategies a) strengthening b) added damping c) base isolation d) weakening only e) full selective weakening (weakening + strengthening)

Selective weakening retrofit intervention for RC frame

Existing literature on the concept of structural weakening ("selective material removal") (FEMA-356, 2000, Pampanin, 2006, ASCE-SEI-41-06, 2007) and weakening with added damping (Viti et al, 2006) were mostly analytical prior to the experimental investigations (Ireland et al, 2007, Jensen et al, 2007) at the University of Canterbury. Conceptually, SW retrofit involves selectively weakening and upgrading certain elements of the structural system to achieve the required hierarchy of strength and deformation capacity.

Consistent with the paradigm shift in seismic engineering to focus on displacement (or material strains) demand-capacity, SW retrofit aims at improving the local inelastic mechanism within the b-c joint subassembly, to a more ductile failure mechanism. By averting joint shear failure or column failures, a ductile beam-sway inelastic mechanism would allow both higher global deformation/ductility capacity and also possibly lower deformation demand due to possible increased damping (fatter hysteresis loop). This can be achieved by purely weakening of the beam as illustrated by Figure 1d where the deformation capacity is extended by the virtue of flexural hinging inelastic mechanism.

However, depending on the as-built b-c joints configuration, there may be need to further upgrade using external post-tensioning in order to achieve the final targeted seismic performance. Joint post-tensioning has been shown to work quite well in reducing the required joint shear reinforcement in bridge-bents (Sritharan et al, 1999) and improving the joint behaviour (Hamahara et al, 2007). Besides, weakening and then post-tensioning allow better control of the desired strength, thus protecting the foundations and other shear-failure sensitive elements within the structure itself.

A more illustrative example of the application of SW retrofit for non-ductile RC frame building is given in Figure 2. By inducing a flexural hinge in the beams by cutting some (or all) of the (bottom) longitudinal beam reinforcement at the exterior b-c joint face, the overall frame, whilst weakened, becomes more ductile, thus achieving a higher deformation capacity (Figure 2b). The amount of weakening permissible is based on the required beam shear-capacity for gravity loading while full hinging under positive moment can be assumed for lateral loading. In a second phase, further strengthening with external post-tensioning can improve the lateral capacity (to the desired limit, below the existing foundation capacity threshold) and/or energy dissipation while still achieving a greater deformation capacity through a more ductile mechanism. This is conceptually shown in Figure 1e and 2c. By adopting a displacement-based retrofit approach, the SW retrofit strategy would become more rationale and clearer.

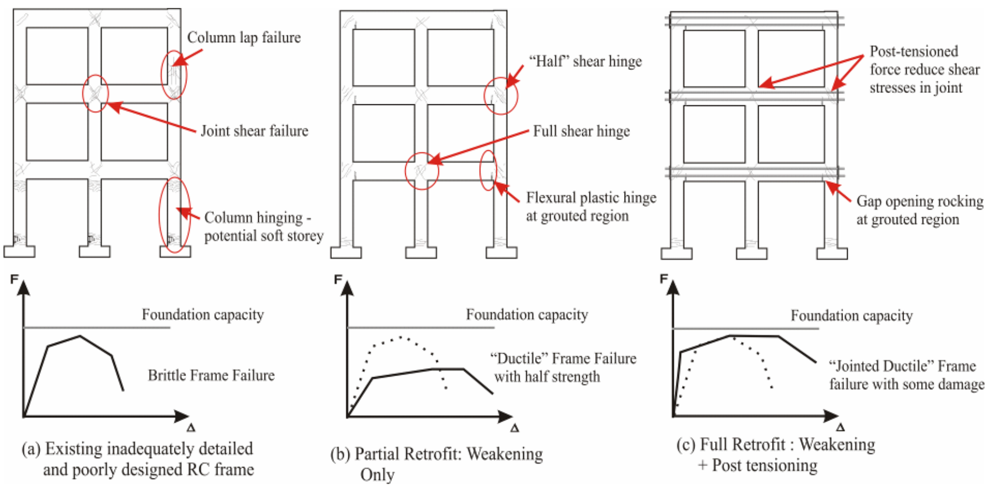

Figure 2. SW retrofit for rc frame: a) existing RC frame b) cutting the bottom longitudinal bars to reduce joint shear stress c) post-tensioning joint and weakened b-c joints.

Experimental and Numerical Validation

Tests description

Nine 2/3-scaled specimens of non-ductile one-way exterior RC b-c joint were tested to validate the SW retrofit concept presented in preceding paragraphs. The as-built b-c joints were designed to be representative of worst-case pre-1970s construction practice while meeting the requirements of older building codes (NZS95:1955, 1955, ACI318-63, 1963). Three as-built configurations were considered – benchmark (NS-O1), benchmark with column lap-splice (S-O1) and benchmark with in-situ slab and transverse beam stub (SL-O1). All test units had 230mm x 230mm (9x9 inch.) columns and 330mm deep x 230mm wide (13 x9 inch.) beams. Geometry and reinforcement detail of the as-built b-c joint are shown in Figure 3.

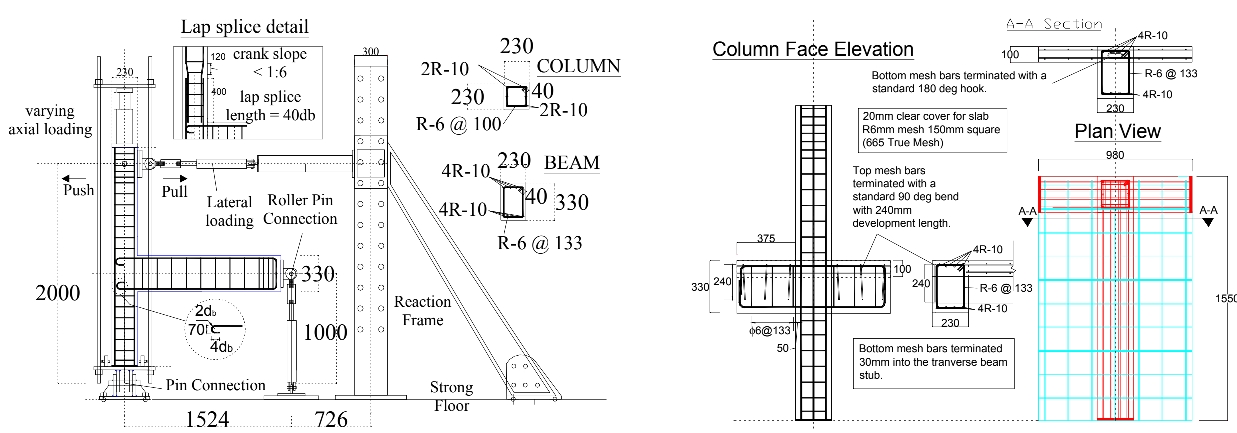

Figure 3. a) Experimental Test Setup and reinforcing detail for benchmark units without slab b) Reinforcing detail for b-c joint with slab (unit shown are in mm).

All the joint cores had no transverse reinforcement and the beams longitudinal reinforcements were anchored into the joint using 180 deg. standard hooks (see Figure 4a). For the column lap-splice, a tension lap of 40d_b was assumed. The cast-in-situ slab has thickness of 100mm with R6 mesh on 150mm square and cantilevered length of 490mm from beam center-line (C/L). The reinforcement detailing of the slab onto the b-c joint was consistent with typical gravity-designed one-way slab, with continuous or tension anchorage for top mesh bars, and discontinued or hooked bottom mesh bars. The transverse beam stub adopted the same reinforcing detailing as the main beam, not an uncommon assumption in pre-1970s construction. Standard steel products were used: mild steel reinforcements and pre-stressing 7-wire 12.7mm diameter strands with average yield strength of 350MPa/50.8ksi (R10), 425MPa/61.6ksi (R6) and 1560MPa/226.3ksi (12.7mm strand).

The remaining six b-c joint specimens investigated the different parameters of SW retrofit including the levels of external post-tensioning forces and locations of beam weakening, in addition to the influence of column lap-splice and cast-in-situ slab and transverse beams. Figure 4b and 4c illustrate the practical implementation of beam weakening and external post-tensioning for the laboratory test specimens. The rationale of the test matrix is described elsewhere (Kam, 2010). The retrofit design was carried out based on the design procedure described in the next section. Brief description of the test units is given in the following Table 1, while further details of the experimental tests can be found in reference (Kam, 2010).

Table 1. Description of b-c joint test units.

Test Unit	Description	Beam Bottom Reinforcements	Cutting Radius (mm)	Weakened section distance from column C/L (mm)	Post-tensioning Force (kN)	Concrete Strength, f'c (MPa) [1]
NS-O1	as-built benchmark specimen	4-R10	-	-	-	17.3
S-O1	as-built specimen with column lap splice	4-R10	-	-	-	15.1
SL-O1	as-built specimen with slab/transverse stub	4-R10	-	-	-	13.4 & 19.9 [2]
NS-R1	R1 Retrofit - beam-weakening only	2-R10 [4]	80	165	-	25.6
NS-R2	R2 retrofit - external post-tensioned (PT) only	4-R10	-	-	120	28.2
NS-R3	R3 Retrofit - beam weakening and external PT	2-R10 [4]	80	165	40	24.3
NS-R4	R3 retrofit with different cutting distance and lower PT	2-R10 [4]	80	310	24	30.3
S-R3	S-O1 specimen with R3 retrofit scheme	2-R10 [4]	80	165	40	20.7
SL-R3	SL-O1 specimen with R3 retrofit scheme	2-R10 [4]	80	165	40	17.0 & 23.1 [2]

Abbreviation: NS=no column lap-splice; O=as-built; R=retrofitted; PT=post-tensioning; R10 = diameter 10mm plain round bars ; C/L = center li
[1] Concrete strength at the day of testing; [2] Top half of the column and other parts were casted separately. The first value given is the top half of th concrete strength. [3] Selective beam weakening with two outer bottom longitudinal bars severed.

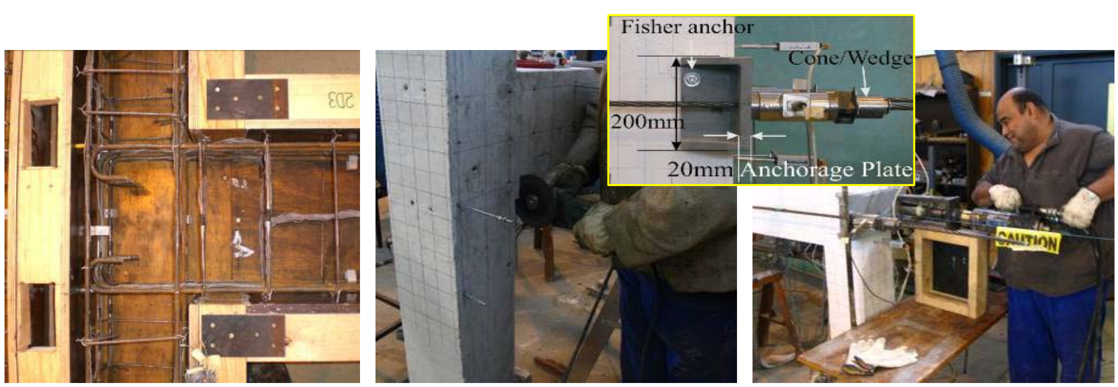

Figure 4. a) B-c joint reinforcing details b) Beam weakening - severing beam bottom

longitudinal reinforcements with plate grinder (arrow) c) Applying external post-tensioning on the exterior b-c joint (insert: anchorage for post-tensioning).

The lateral loading protocol used in this experiment consisted of two displacement-controlled Pull-Push cycles at increasing amplitudes as follows: 0.1%, 0.2%, 0.5%, 1.0%, 1.5%, 2.0%, 2.5%, 3.0% and 4.0% inter-storey drift. Varying vertical axial load 120kN±4.63V_c (26.98±4.63V_c in kips) was implemented to account for the frame action, where V_c is the lateral force applied at the top of the column. All the specimens were thoroughly instrumented to measure: a) lateral force applied b) displacement at the top of the column c) local deformation components d) strains in the reinforcement and e) manual crack widths.

Test Results

The experimental results are summarized in Table 2 and the associated hysteresis curves for each specimen are presented in Figure 5. The final damage patterns of the test units are given in Figure 6. For the benchmark specimen NS-O1, joint shear failure – with subsequent concrete wedge spalling and column bars buckling governed its premature failure mechanism. For S-O1 specimen, lap-splice failure while limited the joint shear stress demand, it accelerated column longitudinal bars buckling thus leading to significant 2nd cycle degradation and premature failure. With floor slab and transverse beam, positive effect on the post-joint-cracking behaviour of SL-O1 was observed. Despite early joint cracking (1st cycle of 1.0% drift) and predominantly shear-hinging inelastic mechanism, SL-O1 activated a relatively stable, despite thin hysteresis loop. This was predominantly due to added confinement from the torsion-induced slab-flange effect on the b-c joint.

Table 2. Summary of experimental test results

Test Unit	Failure Mode	Peak Lateral Force (kN)	Inter-storey drift at maximum force, θ (%)	Ultimate inter-storey drift, θ (rad) [1]	Inter-storey drift at Joint Cracking, θ (rad) [2]
NS-O1	Joint Shear Failure	+14.7 -18.7	+1.95 -0.93	+1.0%-II	+0.9-I
S-O1	Joint Shear & Lap Splice Failure	+14.1 -16.7	+1.43 -0.98	+2.5%-I	+0.7-I
SL-O1	Partial-confined Joint Shear	+21.2 -16.3	+2.42 -2.45	-3.0%-II	+1.0-I
NS-R1	Beam Flexural, Anchorage	+8.2 -15.4	+0.95 -0.80	-2.5%-II	na [3]
NS-R2	Beam and Column Hinging	+18.4 -25.2	+3.56 -1.96	-4.0%-II	+1.5-I
NS-R3	Beam Flexural Hinging	+17.4 -21.6	+3.93 -3.91	na [3]	-4.0-I
NS-R4	Beam Flexural Hinging	+14.9 -22.6	±4.0	na [3]	+2.0-I
S-R3	Beam Flexural Hinging	+15.9 -21.5	±4.0	na [3]	-2.0-I
SL-R3	Beam and Column Hinging	+21.3 -29.3	+3.94 -2.95	na [3]	na [3]

Positive force, displacement and drift correspond to PULL cycles while negative values indicate PUSH cycles. I=1st cycle; II=2nd cycle
[1] Failure point defined as attained peak force was less than 80% of previous peak force; [2] Joint cracking was observed as the appearance of diagonal shear crack and/or sudden drop of lateral load due shear cracking. [3] No failure/cracking (based on the definition) achieved.

Of the four SW retrofit schemes tested (R1, R2, R3 and R4), R3 and R4 gave the most satisfactory performance, as expected. The beam weakening averted the joint shear failure (as demonstrated by NS-R1), but the added external post-tensioning gave added confinement and axial stresses to the joint. External pre-stressing avoided anchorage failure as observed in NS-R1 (as shown in Figure 6) where the hooks, under compression, pushed out the concrete cover. By merely adding post-tensioning without beam weakening, as shown in NS-R2, might have led to column hinging, which was undesirable. Higher level of added post-tensioning forces in SW retrofit (comparing NS-R3 and NS-R4) generally only increased the positive moment (weakened side) in the Pull direction, as bond slip generally governed the upper bound of Push direction lateral capacity. While an attempt to increase the available anchorage length in the beam failed to achieve the desired outcome (in NS-R4), NS-R4 clearly illustrates the repeatability of the performance of NS-R3, albeit with lower post-tensioning forces and shifted weakened section.

The presence of slab increased by nearly 30% the negative moment contribution in the retrofitted b-c joint with slab (SL-R3). In comparison, in SL-O1, slab/transverse beam only added displacement ductility without increase in lateral strength. This reinforces the assumption that bond-slip of plain-round bar reinforcements governs the upper bound in the Push direction for all the retrofitted specimens. Another limiting factor needed to be considered in the assessment/design is the maximum column moment due to lap-splice capacity. As shown in S-R3, despite the change of the inelastic mechanism, the lower post-yield stiffness (when compared to NS-R3 and NS-R4), as well as the vertical cracking in the columns, had indicated lap-splice failure, despite retrofitted by the external post-tensioning.

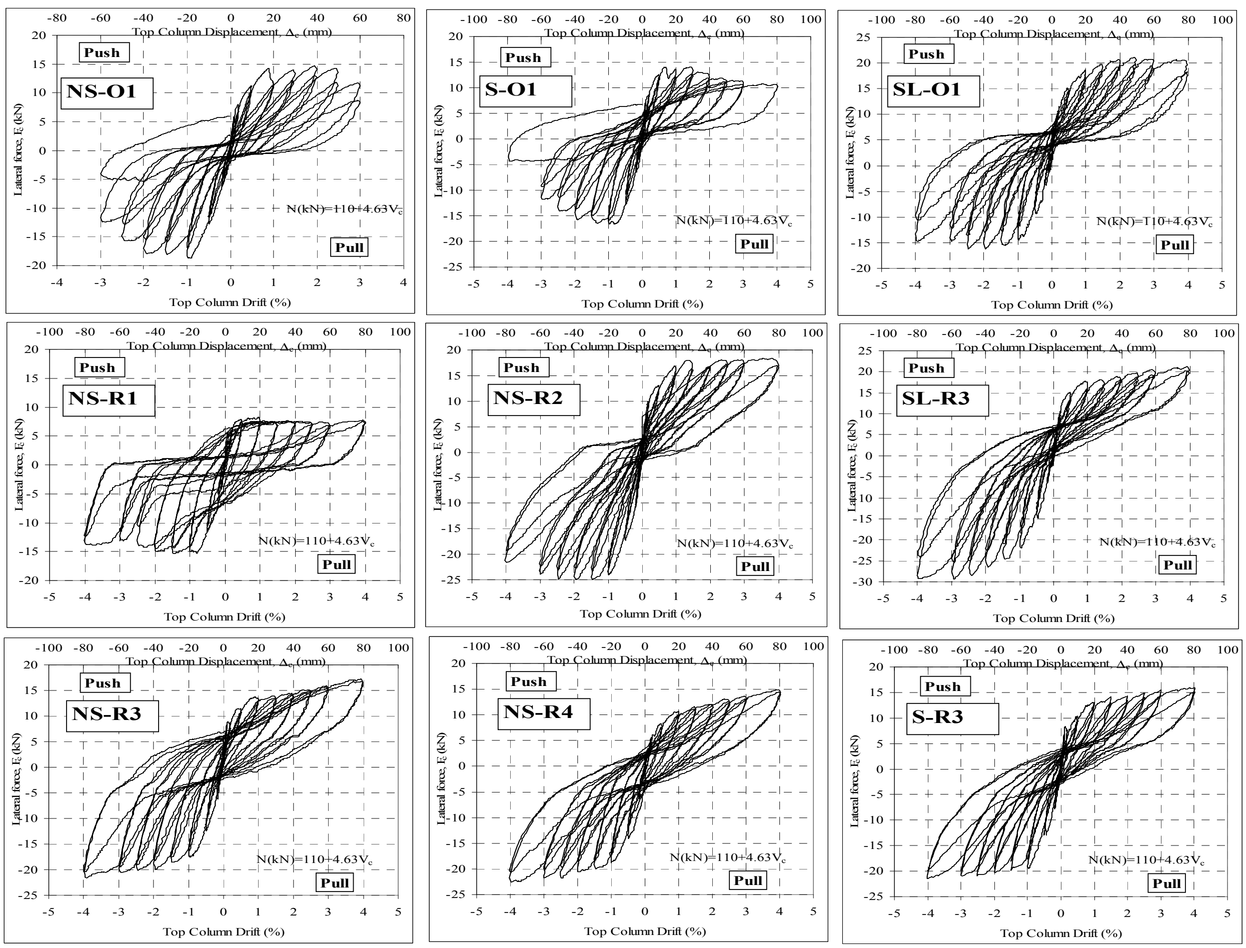

Figure 5. Experimental force-displacement hysteresis curves (1kN = 0.22482kips)

The two intermediate retrofit solutions – beam-weakening only, NS-R1, and external post-tensioning only, NS-R2 - demonstrated the possibility of simple yet efficient retrofit. NS-R1 retrofit was up successful to 2.5% inter-storey drift before failing in compression anchorage push-out. As for NS-R2, partly rocking b-c joint behaviour was attained but limited energy dissipation was achieved due to plain-round bars slipping. Column yielding and hinging beyond 2.5% exacerbated the overall behaviour. The column hinging was activated by the increasing post-tensioning contribution due to gap/crack opening on the b-c interface. As shown in the next section, careful evaluation of the column flexural and shear capacities is necessary if post-tensioning only without beam-weakening is adopted as retrofit scheme (as in NS-R2).

Figure 6. Failure and damage patterns at the end of the tests.

Selective Weakening Retrofit Design Procedure

The design procedure described in this section only considered the local b-c sub-assembly performance domain, with the assumption that a displacement or force-based global assessment procedure would provide the required moment and deformation capacity. By adopting a failure-modes based strength and deformation analysis, the as-built and retrofitted b-c joint can be evaluated within the same M-N (moment-axial load) performance domain. In reality, an iterative assessment-retrofit design-assessment-design adjustment would be required in order to refine the global assessment with the improved damping from changed local inelastic mechanism for the retrofitted RC frames. In-depth presentation of the whole design procedure will be reported in (Kam, 2010).

Seismic evaluation and hierarchy of strength

Different performance limit states can be defined for each structural element of the b-c joints. The elements flexural strength capacities are established using fundamental principles of RC (Paulay et al, 1992). The assessment of shear strength of beams, columns and b-c joints is based on techniques normally employed (Priestley et al, 1996). According to a performance-based assessment procedure proposed by Pampanin (2005, 2006), the concept of principle tensile stresses and empirical joint deformation limit states are used to establish the shear strength and deformation capacities of the joints (see Figure 7c). The hierarchy of strength or better, sequence of events, of a particular b-c connection can then be determined within a M-N performance domain, being (M) the equivalent column moment and (N) the column axial force An example of such assessment for NS-O1 is shown in Figure 7a.

The formulation for the joint shear capacity based on principle tensile (p_t) and compression stresses (p_c) taking into account horizontal (f_h) and vertical (f_v) stresses can be derived for sub-assembly lateral force (V_c):

$$V_c = \frac{V_{jh}}{\left[\frac{l_c}{jd}(1-\frac{h_c}{2l_b})-1\right]} \quad \text{where} \quad V_{jh} = b_{je}h_b\sqrt{\left[p_t^2 - p_t(f_v+f_h)+2f_vf_h\right]} \qquad (1)$$

where l_c = column length, h_c = column depth, l_b = beam length, h_b = beam depth, b_{je} = effective width of the joint. The limiting principle tensile stresses and principle compression stresses can be taken to be $p_t=0.2\sqrt{f'_c}$ and $p_c=0.3f'_c$ respectively for exterior b-c joints with inadequate joint reinforcement, plain round (smooth bars) and poor beam anchorage detailing.

Within the M-N performance domain then, the hierarchy of strength of each elements and the weakest link can then be determined. For example, in NS-O1, joint shear failures were predicted for both Pull and Push directions and were confirmed by experimental results. While not discussed here, it is essential to evaluate all the possible failure mechanisms – including column lap-splice failure and shear failure of flexural elements (column and beam).

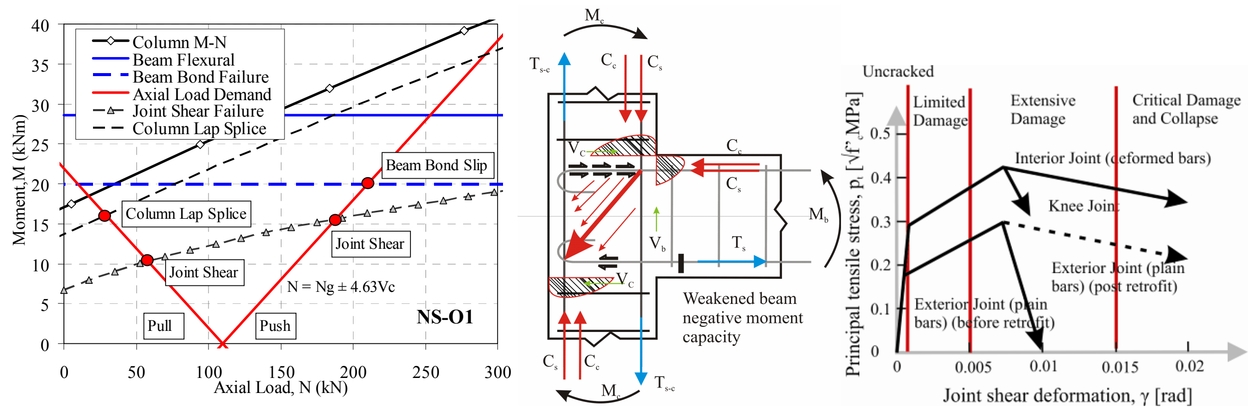

Figure 7. a) M-N Performance Domain for As-built b-c joint - NS-O1 b) Joint shear mechanism (Paulay et al, 1992) and c) joint shear strength degradation model for poorly detailed (e.g. 180deg hooks beam anchorage) b-c joints.

Selective-retrofit intervention – analysis and design

The main aim of the retrofit design is to alter the hierarchy of strength of the b-c joint within the M-N performance domain. In particular, the beam flexural and joint shear capacities after the selected retrofit interventions, e.g. a) beam weakening and/or b) external pre-stressing, need to be evaluated:. While beam flexural-weakening by the means of severing the bottom reinforcement is somehow straight-forward to assess, one must considered the limitation on the available flexural strength due to bond-failure. Assuming a simple triangular bond stress-slip relationship and a uniform bond stress, u_b strength of $0.3\sqrt{f'_c}$ (Fabbrocino et al, 2005), the available flexural capacity of the weakened beam section can be calculated considering the reduction in tensile stress developed in the reinforcement:

$$f_{s-beam} = \frac{4l_d u_b}{d_b} = \frac{4(1.5h_c)(0.3\sqrt{f'_c})}{d_b} \leq f_y \tag{2}$$

where u_b is uniform bond stress capacity, l_b is the development length, d_b is the beam longitudinal beam diameter, f_y is the yield strength of the beam reinforcement. l_b is taken to be $1.5h_c$ to account for tension anchorage from the 180 degree hooks.

External post-tensioning would affect the b-c joint in two significant approach. Firstly, it provides added confinement and horizontal axial stress, f_h to the joint core. This beneficial effect on the joint shear capacity can be quantified by recognizing that $f_h = F_{pti} / (h_b b_{je})$ and that confined exterior joint has a different shear-degradation curve reflected in Figure 7c. Secondly, the beam with added external post-tensioning must be assessed in order to accurate determine the hierarchy of strength (to avoid column hinging). As a first step existing equations available in literature for un-bonded post-tensioned beam can be used to approximate the flexural capacity of the beam:

$$f_{pt-beam} = \frac{0.8F_{pti}}{A_{pt}} + 70 + \frac{f'_c}{100\rho_{pt}} \leq f_{y,pt} \tag{3}$$

where $f_{pt\text{-}beam}$ is the stress in the tendon at nominal flexural strength, F_{pti} is the initial post-tensioning force in the tendon, A_{pt} is the area of the post-tensioning tendon, ρ_{pt} is the ratio of post-tensioned reinforcement = $A_{pt} / h_b b_b$.

Conclusions

This paper has presented the concept, validation and design procedure/example for a novel retrofit strategy for non-ductile RC frames. By a) selectively weakening the beam of exterior joints (NS-R1), b) upgrading the b-c joints using external pre-stressing (NS-R2) or both a) and b) (NS-R3, NS-R4, S-R3), the joint panel zones were protected and an improved inelastic mechanism was activated. In comparison to the benchmark b-c joints with various as-built configurations (NS-O1, S-O1, SL-O1), test results indicated the effective of adopting selective-weakening strategy for the retrofit of b-c joints. In addition, some design equations within the M-N performance domain procedure is presented for the design of SW retrofit of exterior b-c joints. Being economical, non-invasive and low-technology intensity, it is envisioned that SW retrofit could have a wide implementation potential in a macro-scale retrofit scheme.

Acknowledgments

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